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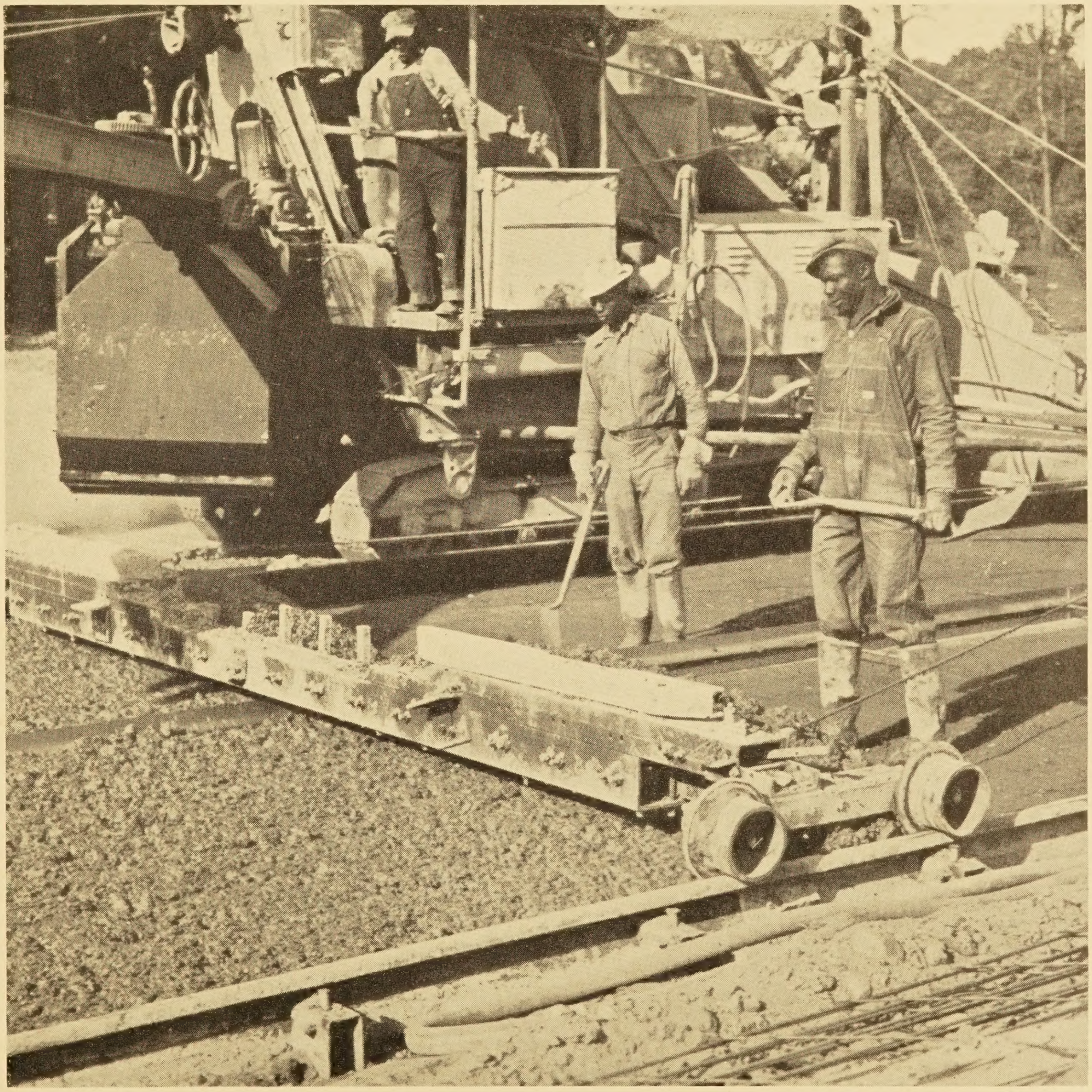

CONSTRUCTING EXPERIMENTAL CONCRETE PAVEMENT IN INDIANA

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to described conditions.

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EXPERIMENTS WITH CONTINUOUS REINFORCEMENT IN CONCRETE PAVEMENTS[1]

Reported by EARL C. SUTHERLAND, Associate Highway Engineer, Public Roads Administration, and SANFORD W. BENHAM, Research Engineer, Indiana Highway Commission.

IN THE FIELD of concrete pavement design, there is probably no subject that has provoked more discussion than that of the proper spacing for transverse joints. Today, after more than 40 years of concrete pavement construction, a wide divergence of opinion still exists as to the proper spacing to use.

The general trend for some years has been toward decreased slab lengths. Gradually the distance between joints has been reduced until at the present time most of the concrete pavement is being laid with slab lengths of 30 feet or less. Theory and experiment have indicated that for satisfactory control of the stresses that are caused by restrained temperature warping a short slab length is necessary.

A pavement designed with short slab lengths obviously contains numerous joints. Joint designs that will fulfill the requirements for flexibility, for ability to transfer load and effectively to control load stress, and for excluding water and foreign matter in a satisfactory manner, have not yet been developed.

There exists an understandable feeling that the trend toward increasing the number of joints required per mile of pavement is a mistake, although present engineering knowledge indicates this to be the most effective design. The cost, the difficulties of installation, the leakage of surface water to the subgrade (sometimes with very harmful results), the tendency for joints to be or to become rough spots producing impact—these and other criticisms are advanced as arguments against the use of short slabs and frequent joints. If it were possible to design a concrete pavement in such a manner that it would be continuous mile after mile, or even if it were possible to space the joints at intervals of 1,000 feet or 500 feet or even less, it is obvious that many of the problems that are associated with frequent joints would disappear. Although this thought has intrigued the minds of engineers for years, the solution to the problem has not been found.

Experience has shown what is to be expected in plain concrete pavements that are laid continuously or with joints placed at infrequent intervals. Contraction and warping stresses cause frequent transverse cracks and lack of provision for expansion may result in "blow-ups". While these troubles have not always been experienced to a serious degree, in general, those interested in the construction and maintenance of highways have felt that the effort to control cracking and other troubles through the use of joints was worthwhile.

The possibilities of pavement slab designs in which the frequency of constructed transverse joints is reduced through the use of continuous, bonded-steel reinforcement have never been very thoroughly explored, although a limited amount of information is available.

Concrete changes in volume when subjected to temperature changes or to changes in its moisture content. A concrete road slab is subject to severe changes in both temperature and moisture content and relatively large volume changes tend to occur. When the slab attempts to contract or to expand, it must overcome the resistance to deformation of the subgrade with which it is in contact. This is the source of the direct tensile and compressive stresses in a transverse section of the pavement. In addition to direct stress, temperature and moisture differentials down through the slab develop periodically and these create bending stresses because of the restraint to warping that exists in slabs of appreciable length. The summation of the stresses that are caused by these conditions of temperature and of moisture, either alone or in combination with the stresses that are caused by vehicle wheel loads, may exceed the tensile strength of the concrete and cause a rupture of the pavement slab.

The introduction of longitudinal steel reinforcement into a concrete pavement will not prevent the formation of transverse cracks under the stress conditions just described. However, the presence of steel will have an important effect upon the character and distribution of the cracking that occurs and upon the structural integrity of the pavement and it is this action that possibly may prove to be of advantage in reducing the number of constructed transverse joints required in concrete pavements.

REINFORCING STEEL EXPECTED TO INFLUENCE THE CHARACTER OF CRACKING

Moderate amounts of steel reinforcement in concrete under tension add relatively little to the tensile resistance of the section prior to the time that rupture of the concrete occurs. Thus, in a concrete pavement, it is to be expected that the first crack will appear at about the same place and under about the same conditions of stress in both a reinforced and an unreinforced slab, other conditions being the same. As soon as rupture of the concrete occurs, however, the concrete section that was most highly stressed is relieved immediately of all stress while any bonded steel that crosses the ruptured section suddenly assumes a much greater burden than before.

The stresses caused by subgrade resistance are carried across the rupture by the steel and transferred gradually back to the concrete through the bond between the two materials. The distance in which the full amount of stress is transferred back to the concrete is dependent directly upon the length of the section and the degree of bond between the two materials and indirectly upon the amount of steel. The result is that in long slabs reinforced with relatively large amounts of longitudinal steel, additional transverse cracks may be expected in close proximity to the first plane of rupture. The distance between the points at which the stress in the concrete section reaches a magnitude sufficient to cause rupture will depend primarily upon the amount of steel that transfers stress across the ruptured concrete section, although for a given percentage of longitudinal steel a certain slab length is necessary to develop the effect that has been described.

[1] Paper presented at the Nineteenth Annual Meeting of the Highway Research Board, December 8, 1939.

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Figure 1.—Present Appearance of a Heavily Reinforced Section of the Columbia Pike Experimental Pavement.

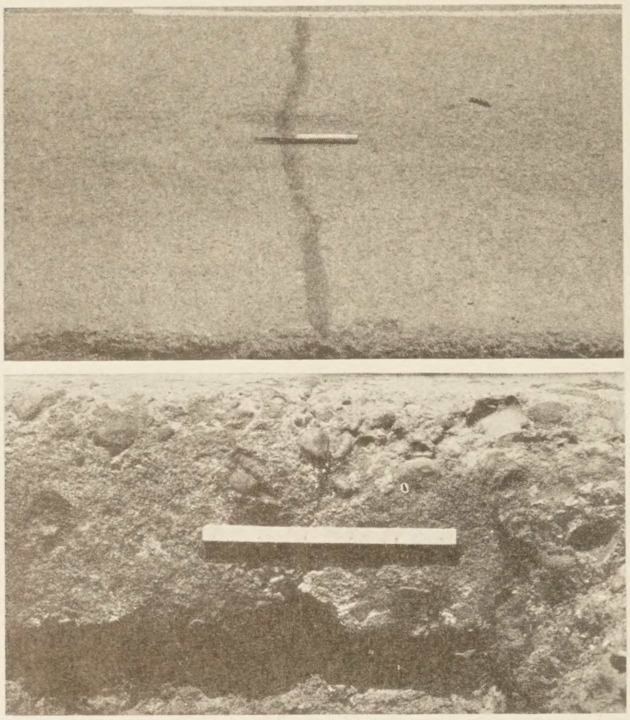

Figure 2.—Typical Crack in a Heavily Reinforced Section of the Columbia Pike Experimental Pavement After 18 Years of Service. Upper, Surface of Pavement; Lower, Edge of Slab.

This action was observed and studied to a limited extent in two of the early researches of the Public Roads Administration, the significant features of which will be discussed briefly.

The earliest of these investigations is that of the Columbia Pike experimental pavement in Arlington County, Va. This pavement, 1¾ miles long, was built in 1921 and included various slab thicknesses, cross-sectional designs, and reinforcement designs. The majority of the reinforced sections are 200 feet long, and the amount of reinforcement in them was varied from a light welded fabric to a heavy bar mat. A published report describes the condition of the pavement at the time it was 2½ years old.[2] The details of design of the various sections of pavement and the characteristics of the subgrade are given in this report. The type of cracking that occurred in relation to the steel reinforcement was discussed in two subsequent papers.[3,4]

The Columbia Pike experimental pavement is of interest at the present time chiefly because it gives some indication of the type of cracking that occurs and the performance that may be expected in pavements containing relatively large amounts of longitudinal reinforcement.

The most heavily reinforced section is 350 feet long and contains 14 three-quarter-inch round deformed bars, uniformly spaced across its 18-foot width. A large number of very fine, closely spaced, transverse cracks developed in the central part of this section soon after the pavement was laid. After 18 years of service, these cracks are still closed and very little spalling or disintegration has occurred in their vicinity. A general view of this section of pavement taken within the last few months is shown in figure 1. Figure 2 shows two views of the pavement at a typical crack. It will be observed that the general condition of this section of pavement is still good and that the structural integrity of the pavement has been maintained. Although figure 1 indicates distances of several feet between transverse cracks, actually it shows only those that have developed to an extent sufficient to catch the eye of the maintenance crew. Additional transverse cracks can be found by close examination, although they are so fine that they are difficult to see, even in a relatively close-up view such as that in figure 2.

The Columbia Pike experiment indicates that in a long concrete pavement slab the presence of relatively large amounts of longitudinal steel will greatly increase the number of transverse cracks but that these cracks will be held tightly closed (if the slab length is not too great) and further that over a long period of time no structural break-down need be anticipated. It shows, also, that the presence of longitudinal steel reinforcement increases the distance between open cracks.

REINFORCEMENT FOUND TO REDUCE CRACKING IN SHORT SLABS AND NEAR ENDS OF LONG SLABS

Several years after the Columbia Pike sections were built, a study of curing methods for concrete pavements was undertaken at the Arlington Experiment Farm, Virginia, utilizing test sections of plain concrete 200 feet long, 2 feet wide, and 6 inches thick. In connection with this study, a number of reinforced sections were included for the purpose of obtaining additional information on certain conditions that had been observed in the experimental sections on the Columbia Pike. The details of the construction and behavior of the various sections included in the curing experiments have been given in two published reports.[5,6]

The amount of steel in the reinforced sections was varied from a very light welded fabric to two ¾-inch round deformed bars (in the 24-inch width). There were, as stated, a number of plain concrete sections and these may be used for comparison with the reinforced sections. Some of the 200-foot sections were duplicated by sections that were divided into five slabs: 20, 30, 40, 50, and 60 feet long. The stresses in the steel and in the concrete caused by the subgrade resistance alone probably would not be excessive in slabs up to 60 feet in length. In the 200-foot sections, however, the stresses caused by the subgrade resistance would exceed the ultimate tensile strength of the concrete, over a considerable part of the total length

[2] Reinforcing and the Subgrade as Factors in the Design of Concrete Pavements, by J. T. Pauls, PUBLIC ROADS, vol. 5, No. 8, October 1924.
[3] The Interrelation of Longitudinal Steel and Transverse Cracks in Concrete Roads, by A. T. Goldbeck, PUBLIC ROADS, vol. 6, No. 6, August 1925.
[4] Concrete Pavement Design, by L. W. Teller and J. T. Pauls, Proceedings American Concrete Institute, vol. 22, 1926.
[5] Tests of Concrete Curing Methods, by J. T. Pauls, PUBLIC ROADS, vol. 7, No. 10, December 1926.
[6] The Arlington Curing Experiments, by L. W. Teller and H. L. Bosley, PUBLIC ROADS, vol. 10, No. 12, February 1930.

of the sections, and would probably exceed the tensile strength of the steel in the more lightly reinforced sections.

In the 13 years since the sections were built, a number of transverse cracks have formed in each of the plain concrete sections and in the central portion of the 200-foot sections that are continuously reinforced with the larger amounts of bonded steel. There has been a negligible amount of cracking near the ends of the continuously reinforced sections and in the reinforced sections that were divided by joints into shorter slabs. This condition obtains in both the sections reinforced with large amounts and those reinforced with small amounts of steel.

In general, no appreciable amount of cracking or disintegration has occurred in the reinforced sections in which the stresses in the steel and concrete, caused by the subgrade resistance, have been held to reasonable values.

In the 200-foot sections containing a relatively large amount of steel, in which the stresses caused by subgrade resistance have exceeded the tensile strength of the concrete, but not the elastic limit of the steel, a number of very fine, closely spaced transverse cracks have developed in the central part of the sections. These cracks have remained closed and little or no spalling or disintegration has developed. A typical crack in one of the reinforced sections containing two ¾-inch deformed bars is shown in figure 3.

These experimental sections have never been subjected to loads of any kind so that the cracking and disintegration that has developed has been caused by the temperature and moisture variations and by other conditions of exposure.

The behavior of the reinforced sections in the curing experiments at Arlington tended to confirm the observations made on the Columbia Pike sections adding support to the theory that has been outlined.

These studies and others that were in progress about the same time emphasized the need for a more thorough understanding of the structural action of concrete pavements and for a number of years attention was concentrated on this problem.

Westergaard developed his important analyses of the stress conditions in pavement slabs under the action of wheel loads[7] and under the influence of temperature changes.[8] The first showed for the first time the relations between load, deflection, and stress for an elastic slab of uniform thickness on an elastic support, dealing with the three critical points, a free edge, a free corner, and an interior point. The second paper, dealing with temperature effects, presented means for calculating the stresses caused by warping restraint. Both of these contributions are of fundamental significance.

In order to study experimentally the relations analyzed by Westergaard and to supplement those analyses by studies of other than uniform cross sections, the Public Roads Administration built and subjected to intensive experiment a series of full-size pavement slabs. This general research covered a period of several years and was divided into a number of parts. Those parts of the investigation that are pertinent to the subject of this paper have been reported and cover the observed effects of temperature and moisture variations on slab behavior and the observed structural action of the several joint designs that were included in the design of the test sections.[9] The slab behavior was determined, in general, by strain measurements.

FIGURE 3.—TYPICAL FINE CRACK IN A HEAVILY REINFORCED SLAB CONSTRUCTED IN THE ARLINGTON CURING TESTS, 13 YEARS AFTER CONSTRUCTION.

RESEARCHES SHOW IMPORTANCE OF CONTROLLING WARPING STRESSES

One of the important results of these researches has been development of an appreciation of the importance of the warping stresses that are present in concrete pavements at times when large temperature differentials are present, stresses that in certain regions of the slab area combine with load stresses to cause critical stress conditions. To control warping stresses, the restraint that produces them must be controlled and this can be accomplished practically by limiting the principal dimensions of the slab. The significance of these combined stresses as they affect the structural design of a pavement has been very completely discussed in a recent paper by E. F. Kelley.[10]

Recognizing on the one hand the experimental evidence that short slabs are necessary for the control of warping and consequently of combined stress and on the other hand the practical objections to the introduction of more joints in concrete pavements, the Public Roads Administration and the Indiana State Highway Commission in 1936 decided to investigate more thoroughly the possibilities of longitudinal steel reinforcement as a means for increasing the slab length of concrete pavements.

Consideration of the problem indicated that the information desired could best be obtained through the construction of special sections of reinforced pavement located on a highway in service and on this basis an extensive research project was planned. Arrangements were then made for the construction of the desired experimental reinforced sections in Indiana as part of a regular Federal Aid project. The State and the Administration cooperated in the selection of a suitable location, the adjustment of the experimental sections to the location, the construction of the sections, and in the program of measurements and observations.

The pavement was constructed on a transcontinental highway west of Indianapolis. This location was

7 Computation of Stresses in Concrete Roads, by H. M. Westergaard, Proceedings of the Highway Research Board, pt. I, 1925.
Stresses in Concrete Pavements Computed by Theoretical Analysis, by H. M. Westergaard, PUBLIC ROADS, vol. 7, No. 2, April 1926.

8 Analysis of Stresses in Concrete Roads Caused by Variations of Temperature, by H. M. Westergaard, PUBLIC ROADS, vol. 8, No. 3, May 1927.

9 The Structural Design of Concrete Pavements, by L. W. Teller and Earl C. Sutherland:
Pt. 2. Observed Effects of Variations in Temperature and Moisture on the Size, Shape, and Stress Resistance of Concrete Pavement Slabs, PUBLIC ROADS, vol. 16, No. 9, November 1935.
Pt. 4. A Study of the Structural Action of Several Types of Transverse and Longitudinal Joint Designs, PUBLIC ROADS, vol. 17, Nos. 7 and 8, September and October 1936.

10 Application of the Results of Research to the Structural Design of Concrete Pavements, by E. F. Kelley, Proceedings of the American Concrete Institute, vol. 35, 1939, also PUBLIC ROADS, vol. 20, Nos. 5 and 6, July and August 1939.

particularly suitable because of the fairly uniform subgrade and the absence of sharp curvature and of steep grades. The maximum grade is 2.95 percent and the maximum curvature is 0°11.5′. Also the traffic on this route is heavy. The test sections were built during September and October of 1938. The remainder of this paper will be devoted to a description of the details of the planning and construction of this pavement, the schedule of observations that has been adopted, and the general behavior of the experimental sections thus far.

In selecting the range in the amounts of steel to be used in the various sections, it was necessary to consider several factors. The maximum amount of steel that can be used will probably be determined by the cost of the steel that can be added and still allow concrete to compete with other types of pavement. There is also a physical limitation to the amount of steel that can be introduced without seriously interfering with the effective placement of the concrete. In order that the steel be adequately protected from corrosion, it must be concentrated in about the middle third of the slab depth. Another factor that influences the distribution and hence the amount of longitudinal steel that can be used is the necessity for some degree of flexibility or hinge action at the transverse cracks for the relief of warping stress.

The minimum amount of longitudinal steel that should be considered is that which might be expected to produce some definite change in the character of the cracking as compared with an unreinforced pavement.

With the data at present available neither the upper nor lower limits can be selected with any degree of precision. For the present study the minimum amount of longitudinal steel was set at No. 6 wire (0.192-inch diameter) spaced at 6-inch centers (22 pounds of longitudinal steel per 100 square feet of pavement). This is equivalent to a 32-pound welded wire fabric, which is about as light as is used in pavements today.

The maximum selected was 1-inch diameter bars on 6-inch centers. This amounts to 534 pounds per 100 square feet and exceeds the amount in any known pavement installation. Between the upper and lower limits of longitudinal steel, intermediate quantities were chosen to give a fairly uniform range of reinforcement. The transverse steel used was the practical minimum thought necessary to secure a good installation of the longitudinal steel.

THREE TYPES OF STEEL USED IN REINFORCING SECTIONS

The number and length of the sections included in the pavement, together with certain details concerning the reinforcement, are given in tables 1, 2, and 3. It will be noted that the sections of pavement included in table 1 are reinforced with welded fabric; those of table 2 with billet steel (intermediate grade), and those in table 3 with rail steel. The three types of steel were included in order that any possible advantages of high elastic limit steel, for the type of construction being considered, might be revealed. The range in steel stress for each class of steel was selected to cover what was considered to be the most effective range. Attention is called to the fact that the range of maximum steel stresses is such as to permit direct comparisons of structural action to be made between sections containing different types or different percentages of longitudinal steel.

The lengths of the slabs necessary to give the desired stresses in the steel were calculated on the basis of the estimated stresses caused by subgrade resistance as the pavement expands and contracts. A coefficient of subgrade resistance of 1½ times the weight of the pavement was assumed in making these determinations.

An analysis of the stress conditions in a longitudinally reinforced section during a temperature drop can be made with reasonable certainty up to the point when the concrete fails and transverse cracks are formed. Because of the uncertainty of the distribution of localized stress in the steel in the vicinity of the cracks it is doubtful if a rigorous analysis can be made of the distribution of stress along the steel after the concrete has ruptured.

Steel stresses calculated on the assumption that the stress is due entirely to the effort of the contracting steel to drag with it along the subgrade the various segments of the fractured section are likely to be both approximate and conservative. For the present purpose the lengths of the sections that are presumably required to develop given steel stresses have been cal-

TABLE 1.—*Details of steel reinforcement in experimental reinforced concrete pavement;* [1] *cold drawn wire (welded fabric)*

Number of sections	Length of each section	Calculated maximum stress in steel	Reinforcement, size and spacing: Longitudinal	Reinforcement, size and spacing: Transverse	Weight of longitudinal steel
	Feet	*Pounds per square inch*			*Pounds per 100 square feet*
149-POUND					
6	140	25,000	No. 4-0; d=0.3938 inch; 4 inches center to center.	No. 3; 12 inches center to center.	132
6	190	35,000			
6	250	45,000			
6	310	55,000			
107-POUND					
6	90	25,000	No. 4-0; d=0.3938 inch; 6 inches center to center.	No. 3; 12 inches center to center.	91
6	130	35,000			
6	170	45,000			
6	200	55,000			
91-POUND					
6	80	25,000	No. 3-0; d=0.3625 inch; 6 inches center to center.	No. 4; 12 inches center to center.	77
6	110	35,000			
6	140	45,000			
6	170	55,000			
65-POUND					
6	60	25,000	No. 0; d=0.3065 inch; 6 inches center to center.	No. 6; 12 inches center to center.	55
6	80	35,000			
6	100	45,000			
6	120	55,000			
45-POUND					
6	30	25,000	No. 3; d=0.2437 inch; 6 inches center to center.	No. 6; 12 inches center to center.	35
6	50	35,000			
6	60	45,000			
6	80	55,000			
32-POUND					
6	20	25,000	No. 6; d=0.1920 inch; 6 inches center to center.	No. 6; 12 inches center to center.	22
6	30	35,000			
6	40	45,000			
6	50	55,000			

[1] Sections are 10 feet wide.

TABLE 2.—*Details of steel reinforcement in experimental reinforced concrete pavement;*[1] *billet steel bars (intermediate grade—deformed)*

Number of sections	Length of each section	Calculated maximum stress in steel	Reinforcement size and spacing: Longitudinal	Reinforcement size and spacing: Transverse	Weight of longitudinal steel
	Feet	*Pounds per square inch*			*Pounds per 100 square feet*
2	360	15,000	1-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	534
2	600	25,000			
2	840	35,000			
2	1,080	45,000			
4	200	15,000	¾-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	300
4	340	25,000			
4	470	35,000			
4	610	45,000			
4	90	15,000	½-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	134
4	150	25,000			
4	210	35,000			
4	270	45,000			
6	50	15,000	⅜-inch round bars; 6 inches center to center.	⅜-inch round bars; 24 inches center to center.	75
6	80	25,000			
6	120	35,000			
6	150	45,000			
6	20	15,000	¼-inch round bars; 6 inches center to center.	¼-inch round bars; 12 inches center to center.	33
6	40	25,000			
6	50	35,000			
6	60	45,000			

[1] Sections are 10 feet wide.

TABLE 3.—*Details of steel reinforcement in experimental reinforced concrete pavement,*[1] *rail steel bars (deformed)*

Number of sections	Length of each section	Calculated maximum stress in steel	Reinforcement size and spacing: Longitudinal	Reinforcement size and spacing: Transverse	Weight of longitudinal steel
	Feet	*Pounds per square inch*			*Pounds per 100 square feet*
2	600	25,000	1-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	534
2	840	35,000			
2	1,080	45,000			
2	1,320	55,000			
4	340	25,000	¾-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	300
4	470	35,000			
4	610	45,000			
4	740	55,000			
4	150	25,000	½-inch round bars; 6 inches center to center.	½-inch round bars; 24 inches center to center.	134
4	210	35,000			
4	270	45,000			
4	330	55,000			
6	80	25,000	⅜-inch round bars; 6 inches center to center.	⅜-inch round bars; 24 inches center to center.	75
6	120	35,000			
6	150	45,000			
6	180	55,000			
6	40	25,000	¼-inch round bars; 6 inches center to center.	¼-inch round bars; 12 inches center to center.	33
6	50	35,000			
6	60	45,000			
6	80	55,000			

[1] Sections are 10 feet wide.

culated in this manner, and a range of stresses selected so that the section lengths which correspond would cover a range that is believed to be sufficient to develop the data desired for each percentage of steel.

It was intended that in each group the longest section would have sufficient length to approach the elastic limit of the steel and that the shortest one would be short enough not to overstress the steel and consequently would develop no open cracks. The condition of the slabs of intermediate length should indicate what maximum length might be used for each percentage of steel with a reasonable assurance that such cracks as occurred would be held tightly closed.

For practical reasons, the standard pavement cross section used by the State of Indiana was adopted. This is a 9—7—9-inch thickened edge section 20 feet wide. The longitudinal joint used was of the deformed metal tongue-and-groove type with ⅝-inch diameter tie bars spaced at 60-inch intervals.

THREE TYPES OF JOINT USED

Because of the wide range and unusual lengths of the sections in this pavement, several different joint widths were selected. It was not possible to predict the amounts of movement that would occur at the ends of the long and the intermediate length sections because the degree of restraint to longitudinal movement of the pavement developed by the subgrade could not be accurately predicted. Three types of joints having different joint widths were designed, however, using the best information available in caring for expansion and contraction of the various section lengths.

The type I joint is of a structural steel design similar to that frequently used at the approaches of bridges. It is designed to allow a 1½-inch movement in each direction and is used at points where two sections of intermediate length are joined.

The type II joint is really two type I joints, spaced 10 feet apart. This allows an effective opening of practically twice that of the type I joints. This type is used where two of the longer sections are joined.

The type III joint is of the conventional dowel type and is used between the shorter sections.

These three types of joints are described in more detail and illustrated with photographs in a later section of this report.

Four sections, each having a total length of 500 feet, were included in which special joint designs and methods of reinforcing were employed. The details of design of these sections are as follows:

No. 1.—Submerged type weakened-plane joints placed at intervals of 10 feet. Reinforcement consisted of a 91-pound welded fabric placed continuously through the weakened-plane joints. The bond between the steel and the concrete was broken for a distance of 18 inches on each side of each weakened-plane joint by omitting the transverse steel at this point and by greasing.

No. 2.—This section was a duplicate of section No. 1, except that it was reinforced with a 45-pound welded fabric.

No. 3.—Weakened-plane joints formed by grooves at 10-foot intervals in the surface of the pavement. Reinforcement consisted of a 91-pound welded fabric, placed continuously through the weakened-plane joints. The bond was broken foı a distance of 18 inches on each side of each weakened-plane joint, as in section No. 1.

No. 4.—This section was a duplicate of section No. 3, except that it was reinforced with a 45-pound welded fabric.

The amount of longitudinal steel in the 91-pound welded fabric is 77 pounds while that in the 45-pound fabric is 35 pounds. The types of weakened-plane joints used in this part of the pavement are shown in figure 4.

The object of this pavement design is to control cracking and to eliminate, as far as possible, warping stresses in the pavement. Breaking the bond between the steel and the concrete over a 36-inch length at the joints permits warping of the pavement slab to take place more freely when the joint edges of adjoining

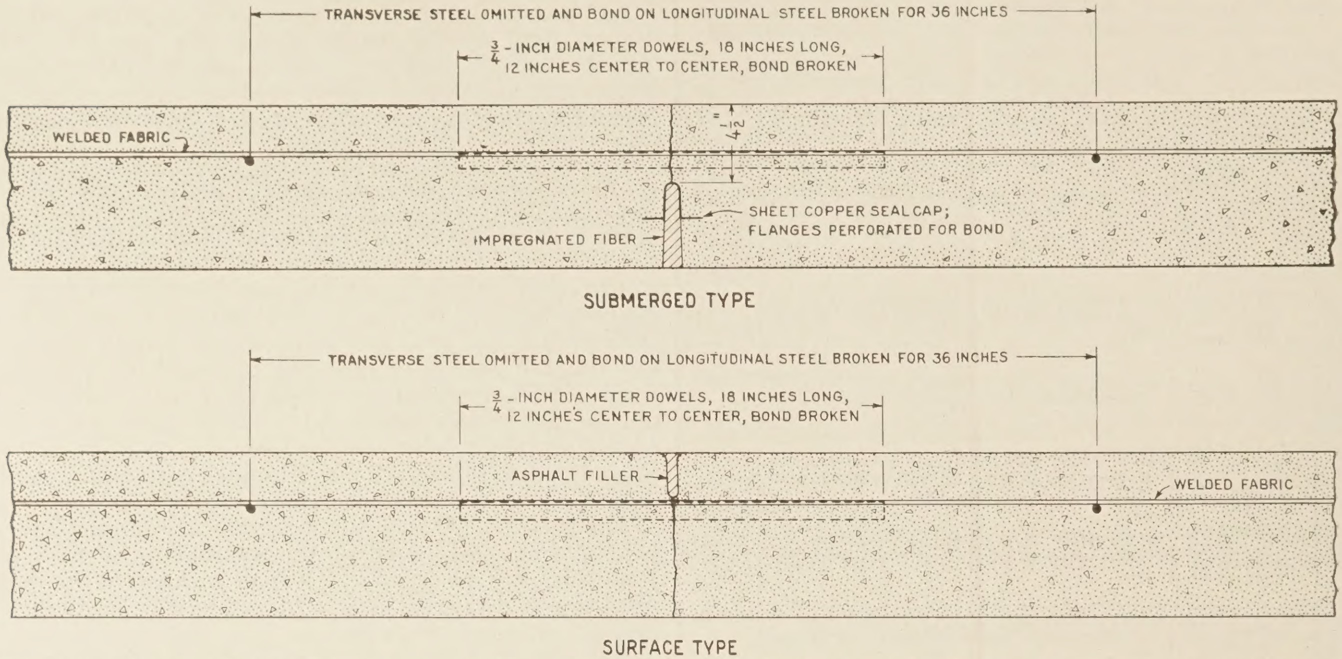

Figure 4.—Details of Weakened-Plane Joints. Dowels Omitted in Joints in One-Half the Length of Each Section.

slabs are in contact, because when warping occurs, a greater length of steel is available to provide the necessary elongation without the development of excessively high tensile stresses in the steel.

The stress conditions that may develop in steel reinforcement passing continuously through several slab units, as in this design, are:

1. That caused by subgrade resistance as the whole slab series expands or contracts about its group center.
2. That caused by the tendency of each short slab unit to expand and contract about its own center. When this occurs, a resisting or opposing force is set up between adjoining slab units causing a change in tensile stress in the steel at the point of connection.
3. That caused by resistance to bending at the joints as the edges of the pavement are warped or are deflected by the action of wheel loads. This stress is the result of the resisting moment set up between the steel and the ends of the slab in contact at the joint as deflection occurs.
4. The shearing stresses caused by loads passing over the joints.

SURFACE VIBRATION USED IN PLACING ALL SECTIONS

It is not possible to calculate accurately any of these stress conditions with the knowledge now available. The first stress condition is understood better than the others and is known to be important in long sections. It is thought that the second and third stress conditions are relieved by the breaking of the bond at the joints, as was done deliberately in that part of the experimental pavement just described. This relief is effective because only a limited opening of the joint at the point where the steel is located is necessary for its accomplishment. If the length of the steel that is available for elongation is increased, there is a corresponding reduction in the intensity of the stress in the steel because the movement at the joint remains the same.

There is no practical means of either computing or measuring the shearing stresses in the steel at weakened-plane joints of the type included in this part of the pavement. It is possible that aggregate interlock will relieve these stresses to a certain degree, but whether this will be sufficient to prevent failure of the steel is not known. For this reason, it was decided to place shear bars across the weakened-plane joints in one-half the length of each of the four sections. If it is later found that a greater number of failures occur in the steel at the joints with no shear bars than occur in those with shear bars, it will be a reasonable conclusion that shearing stresses are influencing the number of failures which occur in the steel.

It is realized that the stresses that will occur in the steel of these four 500-foot sections will probably be sufficient to cause the steel to break. The sections were designed with this in mind because it was desired to determine the number of breaks that will occur in the steel and the amount of opening that occurs at joints in pavement sections designed in this manner.

Because of the length of the pavement it was desirable to let the construction of it under two separate contracts. The two contractors used slightly different materials and methods. While this is unfortunate in a test pavement, it is doubtful if the differences are sufficient to cause serious difficulty in the interpretation of the data.

Because of possible difficulties in the consolidation of the concrete in the sections containing large amounts of steel, it was decided to apply surface vibration to all sections.

The materials and quantities used in each batch of concrete in the west project, F. A. P. 4 A–2 were as follows: Cement, 564 pounds; sand (wet), 1,336 pounds; stone (small), 1,422 pounds; stone (large) 852 pounds.

The slump ranged between ⅞-inch and 1½ inches and averaged 1.3 inches.

The materials and quantities used in each batch of concrete in the east project, F. A. P. 4 B–1 were as follows: Cement, 564 pounds; sand (wet), 1,252 pounds; gravel (small), 1,441 pounds; stone (large) 884 pounds.

The slump ranged between 1 inch and 2¼ inches and averaged 1.5 inches.

The Indiana specifications do not require that the amount of sand be adjusted on the basis of dry weights, but do require that the sand be allowed to stand in stock piles for 48 hours before being used. For this reason, the actual amount of sand used in a batch of concrete is not known.

Attention is called to the fact that crushed stone was used exclusively as a coarse aggregate in the west project, while in the east project the small-size coarse aggregate was gravel and the large-size aggregate was crushed stone. The large-size coarse aggregate ranged in size from ½ inch to 2¼ inches, while the small-size coarse aggregate ranged from No. 4 to 1½ inches.

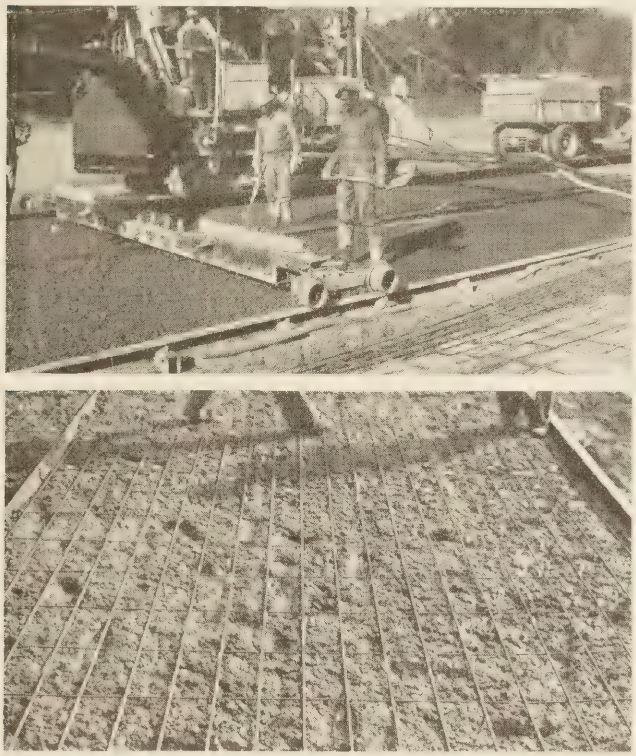

Figure 5.—Construction of Slabs Having Welded Fabric Reinforcement. Upper, Striking Off Concrete Preparatory To Placing Welded Fabric; Lower, Welded Fabric in Place.

The welded fabric reinforcement was placed by the strike-off method at a depth of 2½ inches from the top surface. The method of striking off the concrete preparatory to placing the steel is shown in figure 5. This figure also shows the welded fabric in place before the upper part of the concrete had been placed.

The bar mat reinforcement was erected on the subgrade and securely wired together a short time before the placing of the concrete. This type of reinforcement was placed at the mid-depth of the pavement. It was supported on welded chair assemblies attached to the transverse bars. The three views of figure 6 show different stages in the placement of the 1-inch steel. The supports were welded to every second transverse bar for the ¼- and ⅜-inch longitudinal steel and to every third transverse bar for the larger sizes. This method of supporting the steel proved to be very effective.

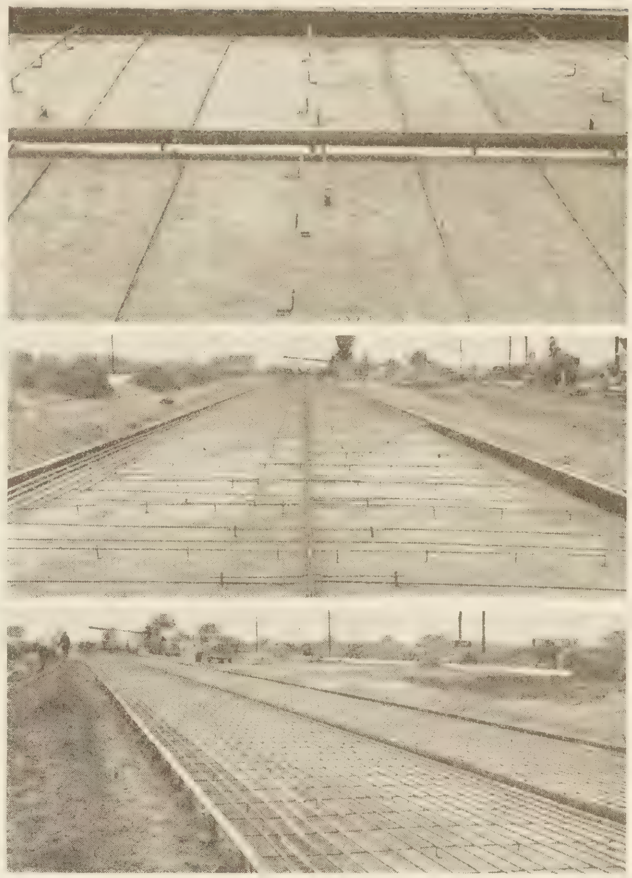

Figure 6.—Installment of Heavy Bar Reinforcement. Top, Welded Chair Supports and Transverse Bars Being Placed; Middle, Longitudinal and Transverse Steel Being Placed; Bottom, Heavy Bar Mat Reinforcement in Place.

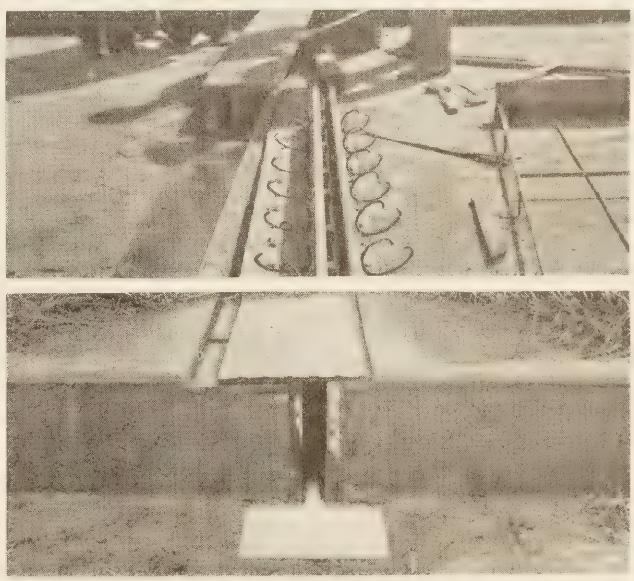

Figure 7.—Type I Joint Being Placed in Position on the Subgrade (Upper), and Appearance of Joint After Construction of Pavement Slabs (Lower).

CONSTRUCTION OF JOINTS DESCRIBED IN DETAIL

The joints used between the experimental sections were described in a general way earlier in the report. The upper view in figure 7 shows a type I joint being

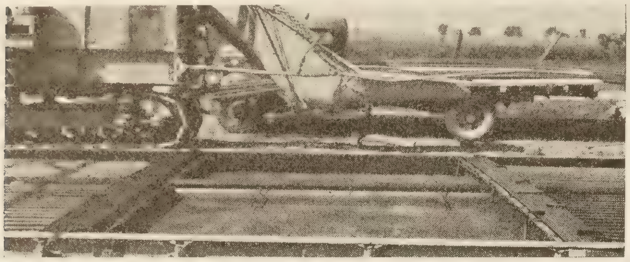

FIGURE 8.—TYPE II JOINT IN PLACE ON THE SUBGRADE PREPARATORY TO PLACING CONCRETE.

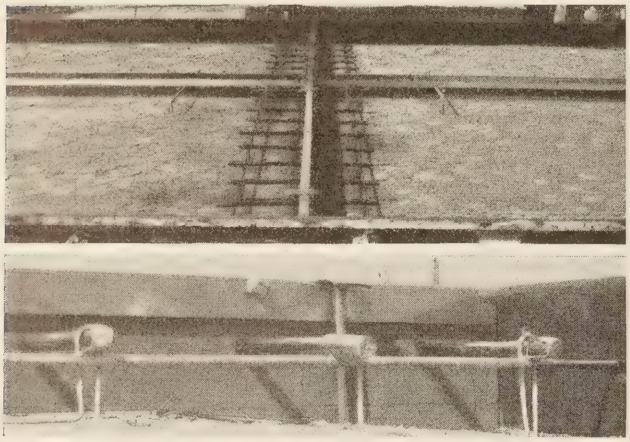

FIGURE 9.—TYPE III JOINT IN PLACE ON THE SUBGRADE (UPPER), AND METHOD OF HOLDING DOWEL BARS IN PLACE (LOWER).

placed in position on the subgrade. The section of joint in the foreground is upside down. This joint consists essentially of two 2-inch angles and a cover plate. The angles are anchored on opposite sides of the joint while the cover plate is rigidly attached to the angle on one side of the joint and held to the angle on the opposite side by a key way system which permits sliding. The different parts of the joint assembly are held together during the time the concrete is being placed by wooden boards that may be removed after the concrete has hardened. The lower view in figure 7 shows an end view of the joint after the concrete has been placed. The projecting part of the sheet metal plate shown on the subgrade is to be turned up against the edge of the slab to close the joint opening before the shoulder is completed.

A type II joint, which consists of two type I joints placed with a 10-foot slab between, is shown in figure 8.

The type III joint is a conventional doweled joint with ¾-inch plain round bars spaced 12 inches apart. A general view of this type of joint and the method of holding the dowels in place are shown in figure 9.

The submerged type of weakened-plane joints were formed by placing impregnated fiber strips on the subgrade at the points where the joints were to be formed. The height of these strips was varied so as to keep the top uniformly 4½ inches below the top surface of the pavement. A copper seal was placed at the top of the groove so that it would be unnecessary to have a watertight seal on the top surface. Figure 10 shows the submerged weakened-plane joint devices in place on the subgrade before placing the concrete. The dowel shear bars which were placed at one-half of the weakened-plane joints were held in place during concreting by tying them to the longitudinal bars of the fabric.

FIGURE 10.—DEVICES USED IN FORMING SUBMERGED WEAKENED-PLANE JOINTS IN PLACE ON THE SUBGRADE.

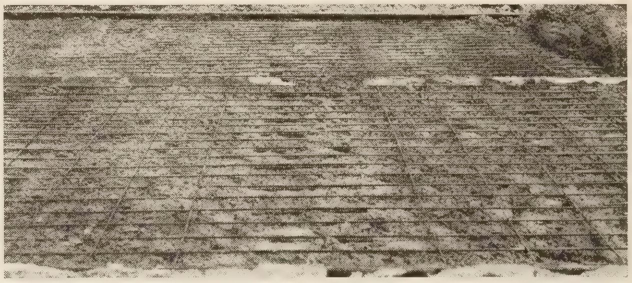

FIGURE 11.—SHEAR DOWEL BARS IN PLACE AT LOCATION WHERE WEAKENED-PLANE JOINT IS TO BE FORMED.

Figure 11 shows the bars in place at one of these joints before placing the top part of the concrete. This method of supporting dowel bars is not recommended for use at joints where longitudinal movements of appreciable magnitude are expected, because of the possibility that some misalinement may develop during the placing of the concrete. In this instance, however, with the 10-foot joint spacing and with longitudinal steel placed continuously through the joints, the longitudinal movements at any one joint should be quite small and with the close supervision given all construction operations it was thought the method of installation would be satisfactory.

Because the vibratory method of placing resulted in dense firm concrete, some difficulty was experienced in forming the grooves in the top of the pavement for the conventional type of weakened-plane joints. This difficulty was satisfactorily overcome by placing a vibrator on the T-bar used to form the groove. This device is shown in figure 12.

The concrete mixer was kept to the side of the roadway during the placing of concrete. This allowed the bar mat reinforcement to be placed a short distance in advance of the mixer. The two views in figure 13 show the placing of the concrete in one of the heavily reinforced sections.

As mentioned previously, surface vibration was used over the entire length of the pavement in order to insure good compaction of the concrete throughout the full depth of the pavement and the proper embedment of the steel. A pan-type vibrator was used on the east project (F. A. P. 4 B–1). The pan was divided in four parts, each approximately 5 feet in length, which were connected to each other by hinged joints. One-third-horsepower vibrators were mounted on each of the four units of the pan and the pan was mounted between the two screeds of the finishing machine. During operation, the pans were in contact with the pavement for a width of approximately 10 inches. The vibrator was operated during two forward passes and

Figure 12.—T-Bar and Vibrator Used in Forming Grooves for Weakened-Plane Joints.

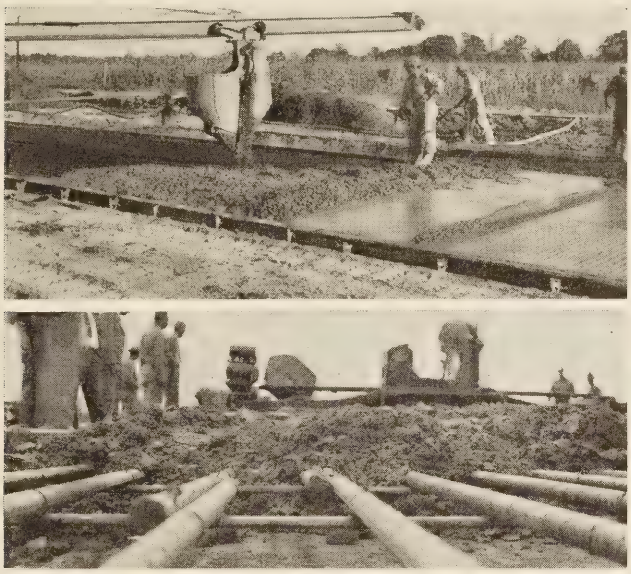

Figure 13.—Concrete Being Placed in a Heavily Reinforced Section.

one backward pass of the finishing machine. The finishing machine with the pan vibrator mounted is shown in figure 14.

On the west project (F. A. P. 4 A–2) the vibration of the concrete was applied through the front screed of the finishing machine to which three ½-horsepower vibrators were mounted. The screed was of the bull-nosed type and vibration was applied to the concrete during two or more forward passes of the finishing machine.

Special vibrators were used around the joints. Two different types of surface vibrators used around the Type III joints are shown in the two views of figure 15. An internal vibrator was used around the type I and type II joints. (See fig. 16.)

The concrete was finished in a conventional way, except that a mechanical longitudinal float was used on the west project. This machine is shown in figure 17.

After placing, the concrete was cured with wet burlap until the next morning. The burlap was then

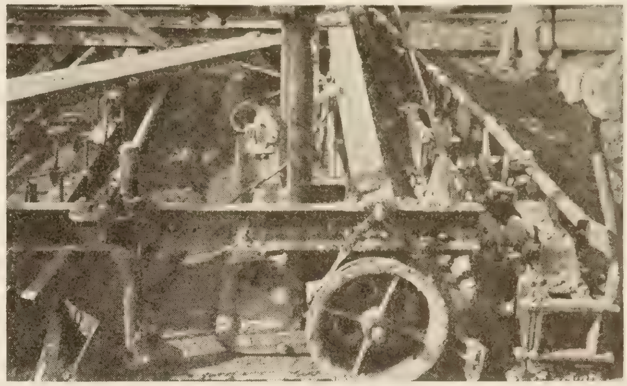

Figure 14.—Concrete Finishing Machine, Showing Pan-Type Vibrator Mounted Between the Two Screeds.

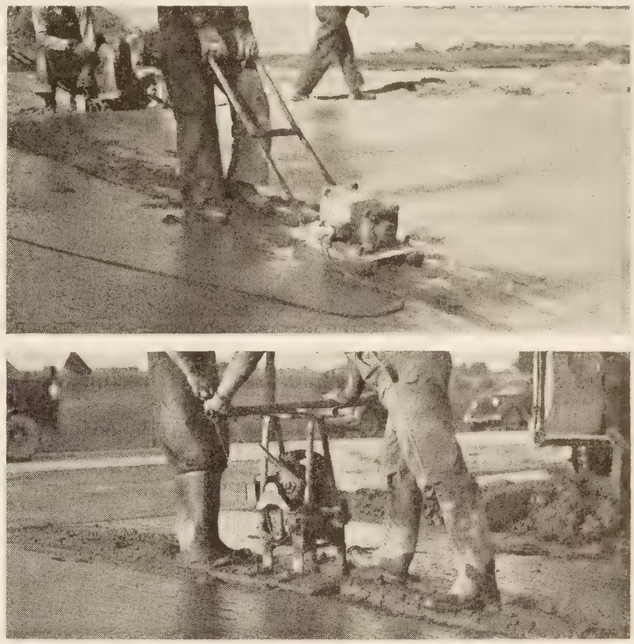

Figure 15.—Vibrators Used Around Type III Joints. Upper, Vibrator Used on East Project; Lower, Vibrator Used on West Project.

Figure 16.—Internal Vibrator Used Under Types I and II Joints.

removed and the concrete was covered with wet straw for seven days. Uniformly excellent weather prevailed during the construction of the entire pavement.

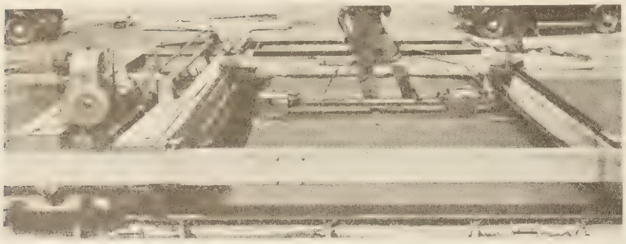

Figure 17.—Longitudinal Mechanical Float Used on the West Project.

SCHEDULE OF OBSERVATIONS OUTLINED

In general, the relative value of the various sections in this experimental pavement can best be determined by a study of their behavior under traffic over a period of years. All of the sections are duplicated at least once and in most cases there are four or six sections of a given type, in order that there will be a check on the performance.

The nature of the pavement makes it necessary that very close examinations be made of the sections if the history of the performance is to be accurately recorded. Transverse cracking is frequently quite inconspicuous, yet from a research standpoint its presence or absence is significant.

The detailed examination of so many sections entails a very considerable amount of work and it is obvious that there are practical limitations to the type and number of surveys that can be made.

Furthermore, weather conditions at different seasons of the year have an important influence on the cracking and other defects that develop in concrete pavements. It is desirable, therefore, that the schedule of periodic examinations be such as to give as much information as possible regarding the seasonal effects.

With these conditions in mind, it was decided to select certain representative sections on which intensive studies would be made at different seasons of the year and to give the entire pavement a thorough general condition survey once a year. This program of course is subject to modification if developments warrant.

For the intensive study just mentioned, three parts of the pavement, each having a length of approximately 2,000 feet, were selected and the details of each are given in table 4.

It will be noted that the first part is located in the heavily reinforced pavement, the second in the pavement having a medium amount of reinforcement, and the third in the pavement reinforced with welded fabric. These different parts of the pavement were located so as to eliminate, as far as possible, all variables except the amount of reinforcement. The intensive schedule of observations which is to be made on these selected sections include the following:

1. A detailed crack survey in which a special effort is made to find all the cracks, however fine.
2. Precise level measurements made on the surface of one lane of the pavement.
3. Measurements of horizontal movements at the joints in the longitudinal direction of the pavement.

These detailed studies of the selected sections are to be made three times a year, in the fall, winter, and spring.

The annual observations made over the full length of the pavement will be made in the fall of the year and include the following:

1. A general crack survey.
2. Measurements of horizontal movements at the joints in the longitudinal direction of the pavement.
3. General observations of the condition of the pavement.

Precise level bench marks were established, at intervals, along one edge of the pavement a short time after the completion of construction. Subsequently level measurements were taken on each side of the joints and at intervals of 100 feet along the full length of one lane of the pavement. This established the normal elevation of the pavement. It is not intended to make further level measurements at regular intervals, but they will be made at such times as may appear to be desirable, either over the full length or on certain parts of the pavement.

Table 4.—*Parts of pavement on which intensive studies are being made* [1]

Part	Length of section	Number of sections	Length of pavement included	Longitudinal reinforcement		
				Type	Size	Spacing of bars
No.	*Feet*		*Feet*		*Inches*	*Inches*
1	600	2	600	Rail	1	6
	360	2	360	Billet	1	6
	1,080	2	1,080	Billet	1	6
2	150	2	150	Rail	½	6
	330	2	330	Rail	½	6
	90	4	180	Billet	½	6
	150	4	300	Billet	½	6
	210	4	420	Billet	½	6
	270	4	540	Billet	½	6
3	20	6	60	32-lb	Welded fabric	
	30	6	90	32-lb	Welded fabric	
	40	6	120	32-lb	Welded fabric	
	50	6	150	32-lb	Welded fabric	
	30	6	90	45-lb	Welded fabric	
	50	6	150	45-lb	Welded fabric	
	60	6	180	45-lb	Welded fabric	
	80	6	240	45-lb	Welded fabric	
	60	6	180	65-lb	Welded fabric	
	80	6	240	65-lb	Welded fabric	
	100	6	300	65-lb	Welded fabric	
	120	6	360	65-lb	Welded fabric	

[1] Sections are 10 feet wide.

Other special studies that are contemplated include:

1. Determination of the movements of the ends of the slabs between the extreme summer and winter conditions; in other words, the maximum annual change in length of the various lengths of slab.
2. Determinations of the daily movements at the ends of certain selected slabs. These measurements will be taken on days when large variations in temperature are expected.
3. Measurement of the absolute movements at the two ends, the center and at the two quarter-points of one of the longest slabs.

The device used in measuring the absolute movements of the ends of the slabs is shown in figure 18. The bench mark, at the right, on which one end of the device is resting, is a pipe that has been driven into the ground to a depth of 9 feet and is surrounded by a casing 28 inches long. There is a small hole in the cap on top of this pipe in which the point of the measuring device is placed. The point on the left end of the measuring device rests in a small hole in a metal plug placed in the concrete. The vernier attached to the point on the left makes it possible to determine the slab movements to 1/100 of an inch. It is necessary to adjust these readings to obtain the actual movements in a direction parallel to the longitudinal axis of the pavement. The bench marks are carefully covered to prevent water from entering and the shoulder material is replaced immediately after taking each reading.

(*Continued on p. 218*)

THE COST OF CURING CONCRETE PAVEMENTS WITH COTTON MATS

BY THE DIVISION OF TESTS, PUBLIC ROADS ADMINISTRATION

Reported by ROBERT A. MARR, Jr., Assistant Testing Engineer

IN CONNECTION with a program initiated in 1936 by the United States Department of Agriculture for the purpose of promoting a greater utilization of cotton products, the various State highway departments were invited by the Public Roads Administration (then the Bureau of Public Roads) to cooperate in a study of the possibilities of using cotton mats for curing concrete pavements.

Laboratory tests by the Administration [1] and field tests by the Texas State Highway Department [2] had already demonstrated the value of this material for curing concrete. Very limited data, however, were available as to the cost of curing with cotton mats compared to other accepted methods. It was felt that accurate data along this line could be obtained only through extensive tests on actual paving jobs located in various parts of the United States.

The State highway departments of 23 States, well distributed geographically, agreed to participate in this program. A total of 90,000 mats, valued at $355,000, was furnished these States by the Federal Government without cost. This quantity would be sufficient to cover at a single placement approximately 65 miles of 20-foot concrete pavement. The mats were made available to the States on the condition that they be used in an approved manner in concrete pavement construction. The cooperating highway departments also agreed to keep records of the life of the mats and average unit curing costs and to report these data to the Public Roads Administration.

In compliance with their agreement, 20 of the 23 participating States have filed reports. These data represent the equivalent of 668 miles of 20-foot pavement cured by this method under widely varying conditions of locale, weather, hourly wage rates, etc., and extending over several construction seasons.

The cotton mats supplied for this purpose weighed approximately 24 ounces per square yard. They were made of cotton bats quilted between cotton sheets of a type known to the trade as "osnaburg." The full-length mats were 22 feet 6 inches long by 5 feet 9 inches wide and were quilted longitudinally by rows of stitching not more than 4 inches apart. For overlap, a 6-inch flap was provided along one longitudinal edge by sewing the covers together without batting. Shorter mats varying in length from 10 feet 6 inches to 14 feet 6 inches were also provided. The mats conformed to Specification M 73–38 of the American Association of State Highway Officials.[3]

Since the mat covers were of unshrunk cloth, considerable change in dimensions was to be expected. Data reported by seven States showed an average change of 6 percent in the longer dimension and 11 percent in the shorter. Cases were reported where individual mats were too short after shrinkage to cover the pavement when laid crosswise. In determining the dimensions of future mats, proper allowance should be made for such shrinkage.

SERVICE LIFE OF 50 USES INDICATED

The specifications provided for these cotton mats to be used in a manner similar to burlap, i. e. that the saturated mats be applied to the concrete as soon as possible without marring and that they be kept wet until removed. No further curing treatment was required. A minimum curing period of 72 hours was specified though several States used a longer period.

The estimated life of an individual mat as reported by 14 States ranged from a minimum of 15 uses to a maximum of 100. The arithmetical average for all States reporting was 47 uses, while a weighted average based on the concrete pavement yardage cured was 63. Ten of the 14 States reporting life data estimated that the mats could be used at least 40 times before discarding them. It seems conservative to state that cotton mats manufactured to American Association of State Highway Officials specifications should have a life of 50 uses with ordinary care, and that this life may be increased 10 to 50 percent by extra care in handling, drying, and storage.

The cost of the mats, as delivered to certain designated points in the States, averaged approximately 48 cents per square yard of net useful coverage.[4] For an average life of 50 uses, this is equal to about 1 cent per square yard. This value checks reasonably well with the average mat cost per use in eight States for mats purchased in the open market of approximately 1¼ cents per square yard.

The cost of curing concrete pavement as revealed by reports from 19 States, excluding the material cost but including supervision, labor, and transportation, averaged 2.15 cents per square yard. These costs were, of course, greatly influenced by local factors, chiefly hourly wage rates and weather conditions. For instance, the minimum State average of 1.08 cents occurred where labor was quoted as low as 20 cents per hour, while the maximum State average of 3.11 cents corresponded to an hourly wage rate of 68 cents per hour. For individual projects the range was from slightly over ½ cent to 6 cents per square yard. On one project, heavy rains made sprinkling unnecessary about one-fourth of the time. This helped to reduce curing costs 40 percent below the average for this State.

[1] Cotton Mats for Curing Concrete. PUBLIC ROADS, vol. 14, No. 5, July 1933. Further Tests of Cotton Mats for Curing Concrete. PUBLIC ROADS, vol. 15, No. 9, Nov. 1934.

[2] Curing Concrete Pavement with Cotton Mats, by J. G. Rollins. American Highways, vol. XIV, No. 3, July 1935.

[3] Standard Specifications for Highway Materials and Methods of Sampling and Testing, published by the American Association of State Highway Officials, 1220 National Press Building, Washington, D. C.

[4] The "net useful coverage" is the actual area of pavement slab which can be covered by a mat of specified dimensions. For instance, after allowing for shrinkage, overlap, and overhanging ends, a mat having gross dimensions of 22 feet 6 inches by 6 feet 3 inches will cover a slab 20 feet by 5 feet in area. The net useful coverage of the mat is, therefore, 100 square feet although its total area is approximately 142 square feet.

FIGURE 1.—COTTON MATS IN PLACE BEING WET DOWN.

The average total curing cost, obtained by adding the average material and usage costs, is 3.15 cents per square yard (1 cent material cost plus 2.15 cents usage cost). However, as certain States omitted such items as cost of water for sprinkling, transportation of mats to and from the project, and the winter storage, it would seem safer to state that the average total cost should not exceed 3.5 cents per square yard. In nine States the cost of curing with cotton mats was compared with the cost of curing with other acceptable materials. A summary of these direct comparisons indicates that the cost of cotton mat curing is in general about the same as other commonly accepted methods used under similar conditions.

All of the State reports submitted in connection with this study confirm the preliminary laboratory tests by showing a high efficiency for cotton mat curing when judged by the following:

1. Ability to maintain a film of moisture over the surface of the concrete during the curing period.
2. Strength of cores from mat-cured slabs as compared to those cured by other standard methods.
3. Insulation of slab against temperature change.

COTTON MATS HAD GOOD ABSORPTION AND RETENTION OF MOISTURE

Although somewhat difficult to wet the first time, due to natural oils in the cotton filler, the mats will absorb from two to three times as much water as double 12-ounce burlap. Figure 1 shows mats in place being wet down. Comments from 11 States reveal that mats retain this absorbed water equally as well as earth and better than burlap. On two projects, mats receiving only the original wetting were still wet on the under side at the end of the 72-hour curing period.

Core tests (based on incomplete data) show that strengths from mat-cured slabs average approximately the same as those cured by other standard methods.

Cotton mats have excellent insulating qualities. Temperature measurements taken on two California projects during cold weather showed that the average minimum temperature under the mats was 40 percent higher than the average minimum air temperatures, of which some were within the freezing range. Even when the top fabric was frozen stiff, the under surface was still soft and moist. Two Northern States also found them very effective in preventing freezing of concrete and subgrade. Limited tests made on a few mats, blackened on top by applying a light coat of emulsified asphalt, showed the average maximum and minimum temperatures under them to be 29 percent and 5 percent higher, respectively, than those under uncolored mats.

Comments by the States reporting, based on experience, indicate that the life of cotton mats can be prolonged by proper attention to certain details, neglect

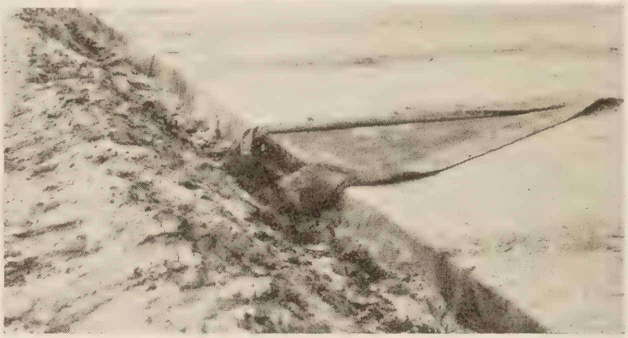

FIGURE 2.—COTTON MATS DISINTEGRATE RAPIDLY IF ALLOWED TO OVERHANG ONTO THE SUBGRADE AS SHOWN.

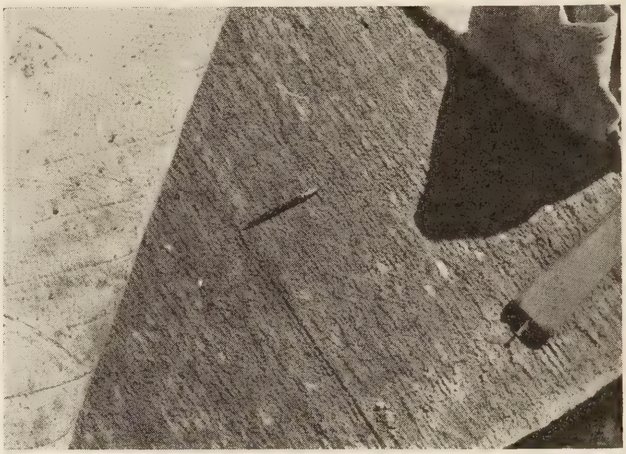

FIGURE 3.—PORTION OF CONCRETE PAVEMENT SLAB AND COTTON MATS USED IN CURING IT.

of which hastens deterioration. A few such precautions are listed below.

If cotton mats are rolled or folded and left damp, even for a few days, mildew sets in, destroying the covering fabric. They must be dried at the end of a job and prior to storage. Under favorable conditions, mats may be left on the slab and turned until dry; but unfavorable weather, particularly in Northern States, may require artificial drying indoors.

Mats should be so stored that they can be periodically inspected for mildew.

When saturated, full-length mats weigh approximately 100 pounds. If the center is allowed to drag on the slab, particularly in removing, this portion of the fabric soon wears out. Full-length mats may easily be handled by two men from a movable bridge. One State has developed a special type of bridge with a sloping apron from which mats can be accurately placed on very soft concrete without marking.

The original intent was for mats to be placed crosswise on the pavement with their ends overhanging the edges of the slab, but constant contact with wet earth was found to cause deterioration. (See fig. 2.) It was found preferable to bank the edges of the slab with earth and to fold the mat ends back even with these edges. Figure 3 shows edges of mats turned back, exposing the concrete surface.

There may be considerable danger from fire in the use of cotton mats, particularly on projects where traffic is maintained in an adjacent lane. Even though the under side is still moist, the top fabric can be dry enough to be ignited by lighted cigarette or cigar butts

(*Continued on p. 219*)

COST OF OPERATING RURAL-MAIL-CARRIER MOTOR VEHICLES

THE report Cost of Operating Rural-Mail-Carrier Motor Vehicles on Pavement, Gravel, and Earth, by R. A. Moyer and Robley Winfrey, has recently been published as Bulletin 143 of the Iowa Engineering Experiment Station. In this report are analyzed cost records of 293 motor vehicles as follows: 248 automobiles operated by rural mail carriers in Iowa, 43 in Indiana, and 2 in Alabama, covering 170 routes.

Operation covered the period from November, 1935 to January, 1937. The report is based upon the original cost records kept by the individual carriers. These detailed daily records covered such phases of operation as miles of travel on each type of road surface, rates of travel, weather, number of stops, load, amounts of gasoline and oil used, and expenses incurred for tires, maintenance, garage, license, taxes, insurance, depreciation, interest, and extra help.

Records submitted were analyzed to determine the average total cost of operation on pavement, untreated gravel, and earth roads for a complete year and for the four seasons. The results obtained apply directly only to cars operating under conditions similar to those encountered by rural mail carriers.

The specific results are summarized as follows:

1. The average operating cost for cars traveling almost exclusively on pavement and gravel was 3.8 cents per vehicle-mile and 7.8 cents per mile for cars traveling almost exclusively on earth.
2. Extra help in delivering the mail cost an average of 0.2 cent per vehicle-mile on pavement and gravel and 1.0 cent per mile on earth.
3. The cost of replacing cars with men on foot or horseback, when the roads were impassable to cars, averaged approximately 11 cents per mile as compared to an average cost of less than 5 cents per mile with the cars when the roads were passable.
4. The graphical solution indicated an average annual mileage of 20,000 miles for cars operated exclusively on pavement and gravel, and 4,000 miles for cars operated exclusively on earth.
5. The average rate of travel (including stops) on the route during the year was about 13 miles in an hour on pavement and gravel, and about 9 miles in an hour on earth. During the summer the approximate average rate on pavement and gravel was 14½ miles in an hour and on earth 10½ miles in an hour, while these rates were respectively 11½ and 7½ miles in an hour during the winter.
6. The cost of gasoline, oil, and maintenance increased from about 2 cents per mile for cars with life mileages under 10,000 miles to 3 cents per mile for cars with mileages of about 50,000 miles. A similar trend was indicated for these costs when the age of the car increased from 1 to 6 years, but there was no appreciable change for cars more than 6 years old.

Results by the statistical method of least squares are:

7. The average cost of gasoline, oil, tires, and maintenance for the year was 1.56 cents per vehicle-mile on pavement, 2.59 on gravel, and 3.14 on earth.
8. The average gasoline mileage obtained was 15.02 miles per gallon on pavement, 13.04 on gravel, and 13.52 on earth.
9. The oil mileage averaged 264 miles per quart on pavement, 159 on gravel, and 113 on earth.
10. During the winter season the cost of gasoline averaged 1.50 cents per mile on pavement, 1.54 on gravel, and 1.58 on earth, while during the summer these unit costs were 1.21 cents on pavement, 1.24 on gravel, and 1.13 on earth.
11. During the winter season the cost of maintenance averaged 0.28 cent per mile on pavement, 0.77 on gravel, and 1.70 on earth, while during the summer season these unit costs were 0.05 cent on pavement, 0.38 on gravel, and 0.63 on earth.

Other results may be summarized as follows:

12. The average factory list weight of the mail-carrier cars was 2,680 pounds and the empty weight was 2,950 pounds as compared to an empty weight of 3,150 pounds for the average Iowa car. The average weight of the mail carried was 135 pounds.
13. The number of boxes per mile of route averaged 4 on pavement and gravel and 3¼ on earth.
14. The total average annual cost of operating the rural-mail-delivery cars, based on an annual mileage of 15,000 miles, was $500.26 on pavement, $627.76 on gravel, and $680.26 on earth, or 3.34 cents per mile on pavement, 4.19 on gravel, and 4.54 on earth.
15. A traffic volume of 63 vehicles per day will justify an annual interest charge of 4 percent on an investment of $1,000 per mile and an increased maintenance expenditure of $40 per year per mile for improving a county trunk earth road with a gravel surface, based on the 0.35-cent-per-mile difference in motor-vehicle operating cost. If an additional charge is made to amortize this investment over a period of 10 years, a traffic volume of 128 vehicles per day will justify the change.
16. A traffic volume of 25 vehicles per day will justify the improvement from earth to gravel if travel time is evaluated as it was for the cars in this study and if the amortization of the investment is included.
17. An expenditure of 0.5 cent per vehicle-mile is justified for snow and ice removal from pavement during the three winter months when the difference in operating cost alone is considered, and 1.22 cents per vehicle-mile is justified when the time factor valued at 40 cents an hour is included.

NEW SUPPLY OF HANDBOOK ON TRANSITION CURVES FOR HIGHWAYS PRINTED

A new supply of Transition Curves for Highways has recently been printed for the Superintendent of Documents, Government Printing Office, Washington, D. C. This handbook, by Joseph Barnett of the Public Roads Administration, contains tables with which the design and location of curves with transitions involve only simple calculations. The rate at which the first supply of this handbook was sold indicates a wide interest in the subject.

Sections of the handbook discuss speed in relation to highway design, design of curves with equal transitions by use of tables, design of curves with transitions as a general case, parallel transitions, transitions for compound curves, adjusting alinement of simple curves for transitions, widening pavements on curves, and right-of-way lines in relation to transitions. All tables needed in applying the methods described are included.

The handbook, in a durable binding, is available only by purchase from the Superintendent of Documents, Government Printing Office, Washington, D. C., at 60 cents a copy. There is no free supply.

The methods described in this handbook are now being used in almost every State and in many foreign countries. Recently the Public Roads Administration approved the use of the handbook by the Argentine Bureau of Roads in preparing a similar publication in Spanish, using metric units and adapted to the Argentine method of laying out curves.

(Continued from p. 214)

Figure 18.—Device Used for Measuring Movements at Ends of Slabs, and Bench Mark (Lower Right).

Figure 19.—Leveling Rod With Special Attachment.

Since the combined movements of the ends of the two slabs at a joint should be equal to the total change in width of the joints, the changes in the widths of the joints are measured as a check on the measurements of the absolute movements of the ends of the slabs. These measurements are made with a micrometer between two metal points set in the concrete.

Metal points were set in the concrete at the points where the level measurements are taken. Figure 19 shows a leveling rod resting on one of these points.

Figure 20.—Typical Crack in a Heavily Reinforced Section.

All measuring points were set slightly below the surface of the concrete so that they would not be disturbed by traffic.

CONSIDERABLE CRACKING IN LONGER SECTIONS AFTER 1 YEAR IN USE

The experimental pavement was approximately 1 year old in October 1939. Approximately 1½ miles had been under traffic for a period of about 1 year while the remaining 4½ miles had been in service for about 6 months. A large number of fine transverse cracks have occurred in the central portion of the long, heavily reinforced sections. In the long sections containing the ¾- and the 1-inch diameter longitudinal bars, the distance between the cracks is frequently less than 3 feet. There is an appreciable number of these cracks in the longer sections containing the ½-inch diameter longitudinal bars, but relatively few in the shorter sections reinforced with this amount of steel. For the sections containing the smaller amounts of reinforcement, in general the number of transverse cracks which have occurred is related more directly with the length of the sections than it is with the amount of longitudinal reinforcement. There is practically no cracking in any of the sections less than approximately 150 feet long, regardless of the amount of reinforcement.

A typical crack in one of the heavily reinforced sections is shown in figure 20. These cracks are not apparent except on close examination and are very similar to those that occurred very early in the life of the heavily reinforced sections of the Columbia Pike experimental pavement. As stated earlier in this report, the cracks in the heavily reinforced sections on Columbia Pike have remained closed and no serious spalling or disintegration has occurred in their vicinity. It seems unlikely, therefore, that these fine cracks in the heavily reinforced sections of this pavement will ever cause serious damage.

The sections in which the weakened-plane joints were placed at intervals of 10 feet are in excellent condition.

Figure 21.—Left, Crack That Has Occurred at a Submerged Weakened-Plane Joint; Right, Weakened-Plane Joint With Surface Groove.

It will be recalled that these sections are 500 feet long and that the steel reinforcement is continuous through the joints. Two of the sections are reinforced with a 91-pound and two with a 45-pound welded fabric. The bond is broken between the steel and the concrete for a distance of 36 inches at each joint.

Cracks have occurred in the surface of the pavement over all except one of the submerged weakened-plane joints. The high stressing of the steel and the breaking of the bond at the joints has allowed these joints to open an appreciable amount. An irregular meandering crack typical of those over the submerged weakened-plane joints is shown in figure 21.

The weakened plane joints of the conventional type, which were formed by placing grooves in the top surface of the pavement, all appear to be in excellent condition. The appearance of a typical joint of this type is shown in figure 21. The manner in which the seal has been maintained at these joints is especially impressive. This tight seal can undoubtedly be attributed to the fact that the short slab lengths and the continuous steel through the joints have reduced to a very small amount the changes in width of the joints caused by the expansion and contraction of the pavement.

(*Continued from p. 216*)

tossed from passing cars. Constant patrolling did not prevent the destruction by fire of 55 percent of all mats in one Southwestern State. Losses by theft can be minimized by stencilling a serial number and ownership.

In conclusion, the data from 19 States indicate that the cost of curing concrete pavements with cotton mats should not exceed that of other accepted methods. The survey also corroborates the laboratory findings that such mats not only retain moisture in the concrete but also have the valuable property of controlling temperatures in the slab, thus providing a type of protection not afforded by the usual surface-sealing materials.

STATUS OF FEDERAL-AID HIGHWAY PROJECTS

AS OF DECEMBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 4,759,921	$ 2,284,401	188.7	$ 5,634,620	$ 2,795,383	194.8	$ 917,390	$ 457,340	25.2	$ 2,201,235
Arizona	1,690,510	1,202,718	78.3	1,765,486	1,251,066	100.5	---	---	---	484,633
Arkansas	4,932,857	3,918,101	226.3	849,062	535,804	37.4	943,598	471,403	33.1	303,090
California	4,659,289	2,536,210	77.3	4,794,259	2,547,856	82.2	1,597,980	854,300	36.1	1,725,573
Colorado	3,815,457	2,119,751	89.3	1,478,378	828,606	38.1	152,739	50,000	4.7	1,603,084
Connecticut	802,205	395,926	10.0	1,520,108	758,035	15.0	---	---	---	1,279,206
Delaware	625,675	297,212	23.7	1,169,783	582,478	14.7	181,020	90,510	.1	1,024,160
Florida	599,651	296,978	8.1	4,159,320	2,079,435	73.9	1,555,912	777,971	73.0	1,801,106
Georgia	3,670,120	1,807,761	202.8	5,219,357	2,609,679	269.5	2,647,338	1,323,669	77.3	4,571,052
Idaho	2,213,871	1,314,379	113.9	780,350	476,373	49.7	221,232	129,875	28.1	994,131
Illinois	8,604,082	4,291,979	190.2	4,932,952	2,464,487	91.1	3,315,150	1,656,575	68.8	1,242,641
Indiana	3,453,840	1,714,428	70.5	5,168,353	2,577,770	108.7	374,694	187,272	8.5	1,819,732
Iowa	3,929,715	1,826,856	190.7	3,310,255	1,460,088	96.4	601,388	284,500	27.4	592,132
Kansas	3,282,010	1,633,293	174.6	2,251,644	1,124,943	118.7	2,916,543	1,454,724	120.7	3,676,351
Kentucky	2,870,347	1,424,314	98.2	2,257,419	1,127,153	45.0	906,765	453,382	22.9	2,696,361
Louisiana	662,972	328,400	23.9	11,733,431	2,915,603	36.6	1,719,885	842,776	50.6	2,288,599
Maine	2,183,320	1,084,502	52.4	717,470	358,735	16.7	13,240	6,620	.6	304,657
Maryland	1,601,408	765,386	23.9	2,470,873	1,217,055	39.5	330,000	155,500	4.6	1,775,766
Massachusetts	3,134,614	1,564,618	25.1	647,015	322,744	4.5	1,375,898	685,030	9.7	2,453,696
Michigan	4,780,068	2,344,648	115.4	2,636,100	1,223,150	89.2	2,436,100	1,201,968	69.5	1,594,786
Minnesota	4,809,253	2,382,604	347.0	4,384,442	2,172,301	192.2	945,766	472,262	52.1	3,037,406
Mississippi	1,268,800	442,660	68.7	8,971,158	3,459,395	343.3	3,046,050	1,299,320	147.6	910,972
Missouri	3,245,097	1,619,407	139.3	3,433,978	1,674,000	105.5	3,601,713	1,437,193	98.7	3,590,981
Montana	3,084,266	1,745,139	185.9	2,411,593	1,366,493	112.7	799,839	453,668	89.3	3,102,650
Nebraska	3,948,865	1,968,342	316.7	3,960,326	1,924,162	375.9	1,922,545	872,392	237.8	2,272,526
Nevada	1,113,680	953,001	53.6	1,029,617	884,238	48.3	70,884	60,713	4.3	635,701
New Hampshire	821,381	404,233	27.3	640,094	313,308	12.1	86,349	42,787	3.4	968.020
New Jersey	532,620	257,786	3.6	4,472,728	2,234,814	34.7	1,717,280	858,640	15.4	969,544
New Mexico	1,785,561	1,098,079	134.7	1,368,261	845,017	78.3	60,211	37,577	19.2	861,430
New York	8,400,260	4,151,472	165.0	11,263,872	5,503,695	168.6	2,286,690	910,445	21.7	186,832
North Carolina	5,145,090	2,563,935	311.5	4,079,716	2,041,597	200.0	812,960	400,310	39.9	1,082,248
North Dakota	267,630	143,396	41.7	1,250,475	670,099	78.0	2,177,590	1,167,133	238.4	3,364,995
Ohio	5,945,384	2,913,332	84.3	6,065,632	3,009,442	48.3	7,919,680	3,833,755	76.6	3,246,966
Oklahoma	1,845,213	975,461	94.7	2,680,817	1,421,884	78.0	2,735,980	1,425,253	94.5	2,520,061
Oregon	2,197,800	1,313,287	102.4	3,022,433	1,641,282	96.4	835,177	501,160	40.3	645,456
Pennsylvania	9,212,237	4,569,464	106.8	4,957,406	2,323,347	45.9	2,907,642	1,431,178	32.5	2,824,608
Rhode Island	601,970	300,865	7.8	834,354	416,221	8.3	201,660	100,830	2.0	825,972
South Carolina	2,165,870	978,200	81.9	1,146,434	500,809	38.3	1,002,559	456,048	105.7	1,837,626
South Dakota	3,462,070	1,912,922	325.4	2,587,710	1,452,650	288.0	1,741,050	974,540	250.7	2,429,786
Tennessee	3,666,788	1,756,626	88.5	2,601,146	1,300,573	49.9	2,323,982	1,162,991	48.5	2,577,890
Texas	10,483,154	5,152,441	611.3	8,122,374	4,035,603	338.8	6,619,513	3,100,477	289.4	1,885,353
Utah	2,121,204	1,526,922	94.4	735,960	531,685	49.7	233,000	163,920	5.4	550,874
Vermont	737,850	361,494	18.4	717,904	358,762	22.7	73,574	36,765	3.0	312,409
Virginia	2,245,860	1,119,948	75.9	2,043,635	973,984	41.4	1,511,560	748,321	39.9	501,827
Washington	2,180,171	1,122,095	38.4	3,258,253	1,606,809	25.9	659,350	283,900	9.1	251,005
West Virginia	1,572,469	849,275	47.4	1,901,105	941,414	45.3	1,007,278	499,334	22.0	1,676,142
Wisconsin	5,217,356	2,565,431	187.9	4,978,600	2,443,880	153.0	99,163	46,805	2.6	1,653,298
Wyoming	1,494,690	922,143	141.3	1,244,964	776,622	123.7	604,868	381,837	62.1	107,909
District of Columbia	373,200	186,600	2.5	264,124	132,062	2.4	103,000	42,188	.8	126,650
Hawaii	366,881	179,428	4.6	786,412	379,777	13.1	601,757	299,199	9.8	1,033,746
Puerto Rico	661,760	329,805	13.8	1,350,769	668,830	25.6	---	---	---	376,160
TOTALS	153,246,362	79,917,654	5,906.0	160,062,527	77,261,198	4,766.5	70,915,482	34,584,356	2,723.6	80,799,034

U. S. GOVERNMENT PRINTING OFFICE: 1940

STATUS OF FEDERAL-AID SECONDARY OR FEEDER ROAD PROJECTS

AS OF DECEMBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR			UNDER CONSTRUCTION			APPROVED FOR CONSTRUCTION			BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	Estimated Total Cost	Federal Aid	Miles	
Alabama	$ 497,335	$ 242,657	27.8	$ 704,332	$ 251,393	21.8	$ 208,000	$ 103,200	11.1	$ 628,006
Arizona	234,160	168,053	25.9	63,551	45,318	7.5	102,861	61,149	16.0	266,292
Arkansas	814,879	680,644	77.1	192,460	177,987	21.3	51,765	34,812	5.8	123,677
California	1,111,026	588,216	43.5	410,544	222,524	7.9	77,458	42,762	8.6	508,031
Colorado	868,570	475,530	31.3	170,641	95,656	5.3	85,425	12,521	1.9	3,086
Connecticut	163,280	67,660	2.9	110,274	54,954	.2				239,855
Delaware	80,623	39,067	17.5	69,537	34,768	7.8				236,350
Florida	401,617	196,550	14.5	501,929	250,507	17.3	113,046	56,523	1.3	361,465
Georgia	303,338	148,082	39.4	195,242	97,621	19.9	390,372	195,186	45.0	950,010
Idaho	457,974	252,546	45.1	128,663	78,481	12.8				105,847
Illinois	1,287,704	637,468	77.1	997,700	443,550	56.7	861,050	430,525	35.5	168,904
Indiana	892,023	442,650	77.7	299,900	148,746	18.3	112,122	56,050	9.4	641,861
Iowa	274,439	130,917	107.0	1,072,286	484,900	104.0	421,565	200,100	85.4	863,890
Kansas	79,068	39,534	43.9	393,524	196,762	6.1	396,115	198,057	19.7	1,147,592
Kentucky	703,407	219,513	61.4	944,343	288,532	47.1	377,637	116,918	39.6	223,613
Louisiana	769,604	356,350	65.2	207,412	103,706	17.3	192,626	87,037	15.0	285,478
Maine	432,057	215,560	25.4	68,900	33,244	3.5	8,300	4,150	.5	7,660
Maryland	204,891	98,787	16.0	149,696	62,848	6.5	157,000	58,355	8.8	315,691
Massachusetts	373,212	185,203	9.2	284,559	140,915	5.9	190,910	94,705	4.2	368,845
Michigan	1,147,358	563,811	105.6	772,020	386,010	39.7	400,350	200,175	22.5	588,386
Minnesota	704,354	347,853	62.4	460,468	230,234	68.4	223,841	68,041	21.5	1,012,377
Mississippi	176,500	88,250	6.8	954,162	470,746	68.9	188,760	94,380	17.1	497,490
Missouri	805,975	383,820	117.4	609,607	289,288	60.7	148,360	58,237	24.9	536,383
Montana	835,992	474,167	71.6	79,149	44,829	6.6	224,054	127,084	27.1	685,948
Nebraska	787,248	381,890	168.9	542,898	264,268	77.5	171,059	85,530	23.5	285,151
Nevada	161,442	133,635	25.0	108,814	93,565	20.1	83,455	71,207	5.4	43,449
New Hampshire	61,156	29,708	2.3	83,280	39,815	3.1	39,055	17,534		132,907
New Jersey	298,990	146,755	10.2	277,250	138,625	14.2	82,880	41,440	.9	469,373
New Mexico	464,923	285,942	42.1	286,665	98,695	13.8	101,564	63,386	13.1	90,619
New York	1,708,811	846,046	90.5	1,748,035	827,073	45.6	501,500	185,550	10.6	70,635
North Carolina	994,549	497,260	94.6	468,883	237,053	39.1	147,790	73,740	19.8	238,792
North Dakota	115,030	61,606	8.3	37,500	20,100	.1	66,953	35,886	2.7	842,938
Ohio	611,800	304,590	36.6	665,089	339,320	25.6	1,399,900	699,950	49.9	951,492
Oklahoma	99,638	48,414	4.7	485,956	258,569	28.3	576,470	306,728	43.3	713,734
Oregon	613,273	347,752	67.6	210,575	89,530	23.5	78,861	47,227	14.5	254,393
Pennsylvania	2,008,795	989,867	113.8	1,209,868	598,829	42.3	237,638	118,819	8.8	175,855
Rhode Island	93,827	46,890	2.2	81,236	40,618	.2	161,211	72,824	3.4	23,483
South Carolina	562,159	228,890	56.9	100,120	36,467	8.3	331,300	114,000	26.3	208,904
South Dakota	16,550	9,100	4.1	11,056	6,088					1,042,862
Tennessee	811,584	352,928	31.7	146,416	73,208	.7	117,367	58,684	7.5	701,718
Texas	2,064,104	1,016,281	239.0	752,478	359,806	63.7	787,330	389,705	88.3	554,884
Utah	224,185	126,765	38.5	100,365	61,098	6.2	20,755	15,430	2.1	128,769
Vermont	145,522	71,278	5.6	164,292	50,336	6.3	54,115	14,000	1.8	40,849
Virginia	632,234	306,985	65.5	308,590	133,848	8.9	321,770	143,490	23.4	94,528
Washington	576,973	300,487	43.8	274,792	144,118	16.9				223,397
West Virginia	145,150	72,575	8.3	237,515	118,757	12.3	167,223	83,611	10.1	317,823
Wisconsin	898,036	446,948	34.5	321,054	160,190	4.7	379,319	179,040	4.4	464,067
Wyoming	466,828	288,087	26.0	336,904	190,329	23.5	53,943	34,052	15.5	5,517
District of Columbia	98,700	49,350	1.0	18,992	8,996	.2				14,779
Hawaii	89,393	44,391	3.7	370,944	185,976	11.0				89,633
Puerto Rico				224,465	109,130	12.8	55,188	27,140	2.1	60,233
TOTALS	28,370,286	14,477,308	2,397.1	19,414,931	9,317,926	1,140.4	10,868,263	5,178,940	798.3	19,007,521

STATUS OF FEDERAL-AID GRADE CROSSING PROJECTS

AS OF DECEMBER 31, 1939

STATE	COMPLETED DURING CURRENT FISCAL YEAR					UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FUNDS AVAILABLE FOR PROGRAMMED PROJECTS
			NUMBER					NUMBER					NUMBER			
	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	Estimated Total Cost	Federal Aid	Grade Crossings Eliminated by Separation or Relocation	Grade Crossing Structures Reconstructed	Grade Crossings Protected by Signals or Otherwise	
Alabama	$ 930,463	$ 917,059	9	2		$ 362,507	$ 361,884	8			$ 35,800	$ 35,800	1		5	$ 789,346
Arizona						518,061	515,813	4								209,120
Arkansas	184,063	184,017	3			669,853	663,348	3			24,235	24,235	2		8	607,099
California	1,156,964	1,156,419	8		1	1,122,970	960,073	6			58,155	58,155	1			948,940
Colorado	615,740	609,909	5		12	17,217	17,217			3	31,856	28,096			10	796,962
Connecticut						194,056	182,342		1	7	222,418	221,637	3			607,587
Delaware	7,839	7,839	2								2,320	2,320			1	516,132
Florida	417,700	417,700	2			207,047	202,548	2			11,928	11,928			3	1,034,415
Georgia	196,480	196,480	5			481,230	481,230	7	3		299,669	299,669	4	2	21	1,888,641
Idaho	204,578	172,785	3			110,538	110,176	1			129,626	100,000	2			366,886
Illinois	2,633,965	2,576,105	17	4	48	1,047,599	966,227	4		19	1,058,888	891,799	4		37	1,264,707
Indiana	756,162	756,162	3	1	54	545,876	545,876	3		31	291,979	291,979		1	54	708,413
Iowa	516,492	465,606	11		40	549,553	514,250	3		117	519,714	434,825	3		92	687,993
Kansas	933,632	933,632	11		5	479,834	479,834	5			459,902	459,901	7		10	611,296
Kentucky	592,898	588,597	8	4	1	689,484	689,484	6		1	284,875	284,875	2		17	359,677
Louisiana	122,838	122,830	2			824,494	770,989	7			317,665	317,659	10			584,469
Maine	331,672	329,136	3	2		206,701	206,701	2	1		924	924			1	235,425
Maryland	24,510	15,403				240,795	240,795	2		2	351,200	336,200	2		13	635,098
Massachusetts	400,519	399,288	2	2		122,047	122,014	2			14,320	14,320	1			1,711,447
Michigan	554,209	551,869	4	2	21	1,060,490	1,060,490	6		13	92,840	92,840	1	1	10	1,320,343
Minnesota	506,912	480,951	4	4	12	1,453,727	1,452,633	10	2	5	122,930	122,930		1	2	872,900
Mississippi	133,900	133,900	2			477,473	477,473	6			246,200	246,200	4	1		681,016
Missouri	402,019	400,714	2	1		1,150,721	1,150,721	7	1		410,364	385,086	2	4		1,373,439
Montana	850,426	850,426	9			213,154	98,073	3			81,312	80,000	2			227,848
Nebraska	625,227	624,627	19		9	726,276	726,276	6	2	13	105,241	105,241			28	486,168
Nevada	196,253	196,253		2	11	23,243	22,341		1	2						104,088
New Hampshire	100,927	100,459	7		1	107,462	107,426	3			91,061	91,061	1			222,539
New Jersey	141,570	141,570	1		1	745,976	745,976	2	3		133,371	133,371	1			1,156,364
New Mexico	59,805	59,805	2			17,848	17,848			1	61,946	61,946	1			630,719
New York	1,540,834	1,535,486	4	6		2,219,008	2,180,908	9	9		1,218,340	1,156,170	4	4		2,131,819
North Carolina	1,145,914	1,110,814	6	4	36	509,209	509,209	7	2	7	326,205	326,205	2		23	595,015
North Dakota	528,012	479,610	7	1		395,927	395,927	5			25,170	25,170			8	441,991
Ohio	453,690	438,690	7	1	2	1,502,813	1,431,861	8	2		714,510	714,510	5		3	2,477,960
Oklahoma	266,389	265,852	3		32	187,025	187,025	3		11	631,602	630,702	13	1	12	1,591,385
Oregon	40,500	39,002				269,872	268,578	3								316,052
Pennsylvania	255,686	255,686		1		2,185,368	1,973,469	7	3		276,449	276,449	5		2	4,157,624
Rhode Island	442,828	442,828	1	3		7,406	7,406									141,016
South Carolina	358,289	324,907	6	3	7	484,787	473,493	4	4		232,965	232,965	1	3	26	643,085
South Dakota	329,373	329,373	3	2	16	79,265	79,265	1	1	1	47,100	47,100	1			996,477
Tennessee	283,757	283,757		2	5	589,158	589,158	3	2							1,339,595
Texas	1,527,206	1,495,940	15	2	4	2,006,068	1,955,492	15	1	1	752,530	687,473	7		21	1,255,934
Utah	192,113	191,963	3		64	167,368	167,368			57	63,300	63,300			23	174,437
Vermont	34,676	29,864			7	225,102	202,214		2							119,233
Virginia	691,979	599,079	7	3	18	150,211	150,211	3			85,295	85,295	1		2	834,481
Washington	294,179	292,768	3		13	174,519	174,519	1	2		131,647	131,647	1	1	1	317,040
West Virginia	64,417	64,417	2			323,834	308,074	7			11,700	11,700			4	969,214
Wisconsin	883,349	879,905	9		2	858,603	812,506	7	2	9	444,636	422,403	2	1	6	652,989
Wyoming	136,598	136,441	1	1	7						85,910	85,910	1			433,605
District of Columbia						366,812	333,268	1								47,053
Hawaii	52,950	50,320	1			140,452	140,452	3	1		236,535	236,525	2		1	115,323
Puerto Rico	49,040	48,840	1			345,312	343,310	8								426,676
TOTALS	23,169,542	22,685,083	223	53	429	27,554,351	26,573,771	203	45	300	10,744,633	10,266,521	99	20	444	41,817,081